

Chapter 6 Structural Steel

6.0	Structural Steel	6-1
6.0.1	Introduction.....	6-1
6.1	Design Considerations.....	6-1
6.1.1	Codes, Specification, and Standards.....	6-1
6.1.2	Preferred Practice.....	6-2
6.1.3	Preliminary Girder Proportioning.....	6-2
6.1.4	Estimating Structural Steel Weights	6-3
6.1.5	Bridge Steels.....	6-6
6.1.6	Available Plate Sizes.....	6-7
6.1.7	Girder Segment Sizes	6-7
6.1.8	Computer Programs.....	6-7
6.1.9	Fasteners	6-8
6.2	Girder Bridges	6-10
6.2.1	General.....	6-10
6.2.2	I-Girders.....	6-10
6.2.3	Tub or Box Girders	6-11
6.2.4	Special Live Load Criteria for Curved Girders	6-12
6.3	Design of I-Girders.....	6-13
6.3.1	Limit States for LRFD	6-13
6.3.2	Composite Section.....	6-13
6.3.3	Flanges.....	6-13
6.3.4	Webs	6-14
6.3.5	Transverse Stiffeners	6-14
6.3.6	Longitudinal Stiffeners	6-14
6.3.7	Bearing Stiffeners	6-14
6.3.8	Crossframes	6-15
6.3.9	Bottom Laterals	6-16
6.3.10	Bolted Field Splice for Girders.....	6-16
6.3.11	Camber.....	6-17
6.3.12	Roadway Slab Placement Sequence	6-18
6.3.13	Bridge Bearings for Steel Girders.....	6-19
6.3.14	Surface Roughness and Hardness.....	6-19
6.3.15	Welding.....	6-20
6.3.16	Shop Assembly	6-21
6.4	Plan Details.....	6-23
6.4.1	General.....	6-23
6.4.2	Structural Steel Notes	6-23
6.4.3	Framing Plan.....	6-23
6.4.4	Girder Elevation.....	6-23
6.4.5	Typical Girder Details.....	6-24
6.4.6	Crossframe Details.....	6-24
6.4.7	Camber Diagram and Bearing Stiffener Rotation.....	6-24
6.4.8	Roadway Slab	6-24
6.4.9	Handrail Details and Inspection Access	6-25
6.5	Shop Plan Review	6-26
6.99	References	6-26

Appendix 6-A

Figure 6-A-1	Steel Plate Girder Framing Plan.....	6-A-1
Figure 6-A-2	Steel Plate Girder Girder Elevation	6-A-2
Figure 6-A-3	Steel Plate Girder Girder Details	6-A-3
Figure 6-A-4	Steel Plate Girder Field Splice	6-A-4
Figure 6-A-5	Steel Plate Girder Crossframes	6-A-5
Figure 6-A-6	Steel Plate Girder Camber Diagram.....	6-A-6
Figure 6-A-7	Steel Plate Girder Roadway Section	6-A-7
Figure 6-A-8	Steel Plate Girder Slab Plan	6-A-8
Figure 6-A-9	Handrail.....	6-A-9

6.0 Structural Steel

6.0.1 Introduction

This chapter primarily covers design and construction of steel plate girder bridge superstructures. Because of their limited application, other types of steel superstructures (truss, arch, cable stayed, suspension, etc.) are not addressed.

Plate girder bridges are commonly used for river crossings and curved interchange ramps. Typical span lengths range from 150 to 300 feet. Steel girders are also being used where limited vertical clearance requires shallow superstructure depth. They may be set over busy highway lanes with a minimum of disruption and falsework, similar to precast concrete elements. Longitudinal launching of steel framing or transverse rolling of completed steel structures has been done successfully.

English units are the current standard for detailing. Metric units may be used on a case-by-case basis. Widening or rehabilitation plan units should be consistent with the original.

6.1 Design Considerations

6.1.1 Codes, Specification, and Standards

Steel highway bridges shall be designed to the following codes and specifications:

- AASHTO LRFD *Bridge Design Specifications, Third Edition*
- AASHTO *Guide Specifications for Horizontally Curved Steel Girder Highway Bridges, 2003* (use with AASHTO *Standard Specifications for Highway Bridges, 17th Edition*) See 6.2.4 for curved girder live load criteria.
- AASHTO/AWS D1.5M/D1.5:2002 *Bridge Welding Code*
- ANSI/AWS A2.4-98 *Standard Symbols for Welding, Brazing, and Nondestructive Examination*

The following codes and specifications shall govern steel bridge construction:

- WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction*, latest edition
- AASHTO/AWS D1.5M/D1.5:2002 *Bridge Welding Code*
- AASHTO *Guide Specifications for Highway Bridge Fabrication with HPS70W Steel*, latest edition

The following AASHTO/NSBA Steel Bridge Collaboration publications are available to aid in the design and fabrication of steel bridges:

- *Design Drawing Presentation Guidelines*
- *Guidelines for Design for Constructibility*
- *Shop Detail Drawing Presentation Guidelines*
- *Steel Bridge Fabrication Guide Specification*
- *Steel Bridge Fabrication QC/QA Guide Specification*
- *Guide Specification for Coating Systems with Inorganic Zinc-Rich Primer*

6.1.2 Preferred Practice

Unshored, composite construction is used for most plate girder bridges. Shear connectors are placed throughout positive and negative moment regions, for full composite behavior. One percent longitudinal deck steel, placed in accordance with LRFD article 6.10.1.7, ensures adequate deck performance in negative moment regions. For service level stiffness analysis, such as calculating live load moment envelopes, the slab may be considered composite and uncracked for the entire bridge length, provided the above methods are used. See LRFD articles 6.6.1.2.1 and 6.10.1.5. For negative moment at strength limit states, the slab is ignored while reinforcing steel is included for stress and section property calculations. Where span arrangement is not well balanced, these assumptions may not apply.

Plastic design may be utilized for simple span and positive moment regions of medium to long span plate girder bridges. In negative moment regions, plastic design is only economical for short span beams.

Currently, economical design requires simplified fabrication with less emphasis on weight reduction. The number of plate thicknesses and splices should be minimized. Also, the use of fewer girder lines, spaced at a maximum of about 14 feet, saves on fabrication, shipping, painting, and future inspection. Widely spaced girders will have heavier flanges, hence, greater stability during construction. Normally, dropping a girder line will eliminate a web without the need for thickening remaining webs. The increased shear requirement can be met with a modest addition of web stiffeners.

For moderate to long spans, partially stiffened web design is the most economical. This method is a compromise between slender webs requiring transverse stiffening throughout and thicker, unstiffened webs. Stiffeners used to connect crossframes shall be welded to top and bottom flanges. Jacking stiffeners shall be used adjacent to bearing stiffeners, on girder or diaphragm webs, in order to accommodate future bearing replacement. Coordinate jack placement in substructure and girder details.

Steel framing should consist of main girders and crossframes. Bottom lateral systems should only be used when temporary bracing is not practical. Where lateral systems are needed, they should be detailed carefully for adequate fatigue life.

Standard corrosion protection for steel bridges is a three-coat paint system, west of the Cascades and where paint is required for appearance. Weathering steel should be considered for dry, eastern regions. When high performance steel is used and appearance is not critical, a single shop coat of inorganic zinc-rich primer may be considered in coastal regions.

WSDOT does not currently allow the use of steel stay-in-place deck forms.

6.1.3 Preliminary Girder Proportioning

The superstructure depth is initially determined during preliminary plan development and is based upon the span/depth ratios provided in Chapter 2 of this manual. The depth may be reduced to gain vertical clearance, but the designer should verify live load deflection requirements are met. It is office practice to limit live load deflections in accordance with the optional criteria of LRFD articles 2.5.2.6.2 and 3.6.1.3.2.

The superstructure depth is typically shown as the distance from the top of the concrete roadway slab to the bottom of the web. Web depths are generally detailed in multiples of 6 inches.

Interior and exterior girders should be detailed as equal. Spacing should be such that the distribution of wheel loads on the exterior girder is close to that of the interior girder. The number of girder lines should be minimized, with a maximum spacing of 14 feet. Three or more girder lines are considered redundant. If a non-redundant bridge is proposed, approval must be obtained from the Bridge Design Engineer.

6.1.4 Estimating Structural Steel Weights

For the preliminary quantities or preliminary girder design, an estimate of steel weights for built-up plate composite “I” girders can be obtained from Figures 6-1 through 6-3.

These figures are based upon previous designs with AASHTO HS-20 live loads with no distinction between service load designs and load factor designs. These charts also provide a good double check on final quantities.

The weights shown include webs, flanges, and all secondary members (web stiffeners, diaphragms, crossframe, lateral systems, gusset plates) plus a small allowance for weld metal, bolts, and shear connectors.

Composite Welded Steel Plate “I” Girder — Simple Span

Figure 6-1

Composite Welded Steel Plate “I” Girder — Two Continuous Spans
Figure 6-2

Composite Welded Steel Plate “I” Girder — Three or More Continuous Spans
Figure 6-3

6.1.5 Bridge Steels

The most common types of steel used for bridges are now grouped in ASTM A 709 or AASHTO M 270 specifications. The following table shows equivalent designations. Grades of steel are based on minimum yield point.

ASTM	ASTM A 709	AASHTO M 270
A 36	Grade 36	Grade 36
A 572 gr 50	Grade 50	Grade 50
A 992 (W shapes)	Grade 50S	---
A 588	Grade 50W	Grade 50W
	Grade HPS50W	Grade HPS50W
A 852	Grade 70W	Grade 70W
---	Grade HPS70W	Grade HPS70W
A 514*	Grade 100*	Grade 100*
	Grade 100W*	Grade 100W*

* Minimum yield strength is 90 ksi for plate thickness greater than 2 ½”.

Plates and rolled sections are available in these specifications and grades. Rolled sections include beams (W, S, and M shapes), H-piles, tees, channels, and angles. These materials are prequalified under the Bridge Welding Code. The common specification for wide flange beams is now ASTM A 992.

Use AASHTO M 270 grade 50W for plate girders. The fabricated costs of structural carbon and structural low alloy steel plate girders are about equal. AASHTO now recommends grade HPS70W instead of grade 70W for bridge use. HPS70W can be economical if used selectively in hybrid design. For moderate spans consider HPS70W for the bottom flanges throughout and top flanges near interior piers. The use of M 270 grade 100 or 100W requires approval by the Bridge Design Engineer, and should not be used until grade HPS100W is available.

All main load-carrying members or components subject to tensile stress shall be identified in the plans and be required to meet the minimum Charpy V-notch (CVN) fracture toughness values as specified in LRFD table 6.6.2-2, temperature zone 2. Fracture critical members or components shall also be designated in the plans.

Availability of weathering steel can be a problem for some sections. For example, steel suppliers do not stock angles or channels in weathering steel. Weathering steel wide flange and tee sections are difficult to locate or require a mill order. ASTM A 709 and AASHTO M 270 bridge steels are not stocked by local service centers. The use of bridge steel should be restricted to large quantities such as found in typical plate girder projects. The older ASTM specification steels, such as A 36, should be specified when a fabricator would be expected to purchase from local service centers. The older AASHTO designations, such as M183, have been dropped.

Structural tubes and pipes are covered by other specifications. See the latest edition of the AISC *Manual of Steel Construction* for selection and availability. These materials are not considered prequalified under the Bridge Welding Code. They are covered under the Structural Welding Code AWS D1.1. Structural tubing ASTM A 500 is not recommended for dynamic loading applications unless minimum CVN requirements are specified.

6.1.6 Available Plate Sizes

Readily available lengths and thicknesses of steel plates should be used to minimize costs. Tables of standard plate sizes have been published by various steel mills and should be used for guidance. These tables are available through the steel specialist, or online.

In general, an individual plate should not exceed 14 feet in width, including camber requirements, or a length of about 60 feet. If either or both of these dimensions are exceeded, a butt splice is required and should be shown or specified on the plans. Some plates may be available in lengths over 90 feet, so web splice locations should be considered optional. Quenched and tempered plates are limited to 50 feet, based on oven size.

Plate thicknesses of less than $\frac{5}{16}$ inches should not be used for bridge applications.

When metric units are used, all steel dimensions, including thickness, should be hard converted. For example, specify 25 mm, not 25.4 mm plate.

Preferred plate thicknesses, English units, are as follows:

- $\frac{5}{16}$ " to $\frac{7}{8}$ " in $\frac{1}{16}$ " increments
- $\frac{7}{8}$ " to $1\frac{1}{4}$ " in $\frac{1}{8}$ " increments
- $1\frac{1}{4}$ " to 4" in $\frac{1}{4}$ " increments

6.1.7 Girder Segment Sizes

Locate bolted field splices so that individual girder segments can be handled, shipped, and erected without imposing unreasonable requirements on the contractor. Crane limitations need to be considered in congested areas near traffic or buildings. Transportation route options between the girder fabricator and the bridge site can affect the size and weight of girder sections allowed. Underpasses with restricted vertical clearance in sag vertical curves can be obstructions to long, tall segments shipped upright. The region should help determine the possible routes, and the restrictions they impose, during preliminary planning or early in the design phase.

Segment lengths should be limited to 150 feet, depending upon cross section. Long, slender segments can be difficult to handle and ship due to their flexibility. Horizontal curvature of girder segments may increase handling and shipping concerns. Out-to-out width of curved segments, especially box girders, should not exceed 14 feet. Weight is seldom a controlling factor for I-girders. However, 40 tons is a practical limit for some fabricators. Limit weight to a maximum of 100 tons if delivery by truck is anticipated.

Consider the structure's span length and the above factors when determining girder segment lengths. In general, field splices should be located at dead load inflection points. When spans are short enough, some field splices can be designated optional if resulting segment lengths and weights meet the shipping criteria.

6.1.8 Computer Programs

The designer should consult the design supervisor to determine the computer program best suited for a particular bridge type. Instruction manuals for the programs are available in the Bridge Office Computer Section.

Office practice and good engineering principles require that the results of any computer program or analysis be independently verified for accuracy. Also, programs with built-in code checks must be checked for default settings. Default settings may reflect old code or office practice may supersede the code that the program was written for.

6.1.9 Fasteners

All bolted connections shall be friction type (slip-critical). Assume Class B faying surfaces where inorganic zinc primer is used. If steel will be given a full paint system in the shop, the primed faying surfaces need to be masked to maintain the Class B surface.

Properties of High-Strength Bolts

Material	Bolt Diameter	Tensile Strength ksi	Yield Strength ksi
AASHTO M 164	$\frac{5}{8}$ - 1 inc	120	92
(ASTM A325)	$1\frac{1}{8}$ - $1\frac{1}{2}$ inc.	105	81
	Over $1\frac{1}{2}$		Not Available
ASTM A 449	$\frac{1}{4}$ - 1 inc	120	92
(No AASHTO	$1\frac{1}{8}$ - $1\frac{1}{2}$ inc.	105	81
equivalent)	$1\frac{3}{4}$ - 3 inc.	90	58
	Over 3		Not Available
AASHTO			
M 314	$\frac{1}{4}$ - 3 inc	125-150	105
(ASTM F 1554)			
Grade 105			
AASHTO			
M253	$\frac{5}{8}$ - $1\frac{1}{2}$ inc.	150-170	130
(ASTM A 490)			
	Over $1\frac{1}{2}$		Not Available
ASTM A 354	$\frac{1}{2}$ - $2\frac{1}{2}$ inc.	150	130
Grade BD			
(No AASHTO	3 - 4 inc.	140	115
equivalent)	Over 4		Not Available

General Guidelines for Steel Bolts

- A. M 164 (A325) High strength steel, headed bolts for use in structural joints. These bolts may be hot-dip galvanized. Do not specify for anchor bolts.
- B. A449 High strength steel bolts and studs for general applications including anchor bolts. Recommended for use as anchor bolts where strengths equivalent to A325 bolts are desired. These bolts may be hot-dip galvanized. Do not use these anchor bolts for seismic applications due to low CVN impact toughness.
- C. M 314 (F1554) Grade 105 — Higher strength anchor bolts to be used for larger sizes ($1\frac{1}{2}$ " to 3"). When used in seismic applications, ASTM F 1554 shall be specified, since AASHTO M 314 lacks the CVN supplemental requirements. Specify supplemental CVN requirement S5 when these are used in seismic applications (most bridge bearings that resist lateral loads). Lower grades may also be suitable for sign structure foundations. This specification should also be considered for seismic restrainer rods, and may be galvanized.

- D. M 253 (A490) High strength alloy steel, headed bolts for use in structural joints. These bolts should not be galvanized, because of the high susceptibility to hydrogen embrittlement. In lieu of galvanizing, the application of an approved zinc rich paint may be specified. Do not specify for anchor bolts.
- E. A354 Grade BD — high strength alloy steel bolts and studs. These are suitable for anchor bolts where strengths equal to A490 bolts are desired. These bolts should not be galvanized. If used in seismic applications, specify minimum CVN toughness of 25 ft-lb at 40 deg F.

6.2 Girder Bridges

6.2.1 General

Once the material of choice, structural steel has been eclipsed by reinforced and prestressed concrete. Corrosion and fatigue cracking have contributed to unanticipated life cycle costs. Fabrication and material costs have also contributed to steel's relative cost disadvantage. Costs may be minimized by simplifying fabrication details, optimizing the number of girder lines, allowing for repetitive fabrication of components such as crossframes and stiffeners, and ensuring ease of shipping and erecting.

The specifications allow a combination of plastic design in positive moment regions and elastic design in negative moment regions. Plate girders, of the depths typically built in this state, have traditionally been designed to elastic limits or lower. Newer design methods may help reduce steel weight and narrow the cost gap between steel and concrete bridges. High performance steel can be used to advantage saving weight and painting.

6.2.2 I-Girders

Welded plate I-girders constitute the majority of steel girders designed by WSDOT. The I-girder represents an efficient use of material for maximizing stiffness. Its shortcoming is inefficiency in resisting shear. Office practice is to maintain constant web thickness and depth for short to medium span girders. Weight savings is achieved by minimizing the number of webs used for a given bridge. This also helps minimize fabrication, handling, and painting costs. Current office practice is to use a minimum of three girders to provide redundant load path structures. Two girder superstructures are considered non-redundant and hence, fracture critical.

Buckling behavior of relatively slender elements complicates steel plate girder design. Most strength calculations involve checks on buckling in some form. Local buckling can be a problem in flanges, webs, and stiffeners if compression is present. Also, overall stability needs to be insured for all stages of construction, with or without a roadway deck. The art of designing steel girders is to minimize material and fabrication expense while ensuring adequate strength, stiffness, and stability.

I-girders are an excellent shape for welding. All welds for the main components are easily accessible and visible for welding and inspection. The plates are oriented in the rolling direction to make good use of strength, ductility, and toughness of the structural steel. The web is attached to the top and bottom flanges with continuous fillet welds. Usually, they are made with automatic submerged arc welders. These welds are loaded parallel to the longitudinal axis and resist horizontal shear between the flanges and web. Minimum size welds based on plate thickness will satisfy strength and fatigue requirements in most cases. The flanges and webs are fabricated to full segment length with full penetration groove welds. These welds are inspected by ultrasound (UT) 100 percent. Tension welds, as designated in the plans, are also radiographed (RT) 100 percent. Office practice is to have flanges and webs fabricated full length before they are welded into the "I" shape. Weld splicing built-up sections results in poor fatigue strength and zones that are difficult to inspect. Quality welding and inspection requires good access for both.

6.2.3 Tub or Box Girders

Typical steel box girders for WSDOT are trapezoidal tub sections. Using single top flange plates to create true box sections is very uncommon when reinforced concrete decks are used. Tub girders will be referred to herein as box girders, as in LRFD article 6.11.

The top lateral system placed inside the girder is treated as an equivalent plate, closing the open section, to increase torsional stiffness before slab curing. Although not required by the code, it helps ensure stability that may be overlooked during construction. Partial or temporary bracing may be used provided it is properly designed and installed. Dramatic construction failures have occurred due to insufficient bracing of box girders. Stability of the shape must be ensured for all stages of construction per LRFD article 6.11.3. The cured deck serves to close the section for torsional stiffness. Internal crossframes or diaphragms are used to maintain the shape and minimize distortion loading on individual plates and welds making up the box. Box segments will have considerable torsional stiffness when top lateral bracing is provided. This may make fit-up in the field difficult.

The ability to make box girders with high torsional stiffness makes them a popular choice for short radius curved structures. Curved box girders, because of inherent torsional stiffness, behave differently than curved I-girders. Curved box girder behavior is approximated by the M/R method, rather than the V-load method. See curved girder references listed at the end of this chapter for complete description.

Straight box girders, when proportioned in accordance with LRFD article 6.11.2 may be designed without consideration of distortional stresses. The range of applicability for live load distribution is based on:

$$0.5 \leq N_L/N_b \leq 1.5$$

which limits the number of lanes loading each box. Wide box girder spacing, outside this range, will require additional live load analysis. Consideration must be given to differential deflection between boxes when designing the roadway deck. Generally, use of crossframes between boxes is limited to long spans with curvature.

Box girders should be detailed for single bearings per box. If bearings are located under each web, distribution of loads is uncertain. Generally, plate diaphragms with access holes are used in place of pier crossframes.

With the exception of effects from inclined webs, top flanges and webs are designed as if they were part of individual I-girders.

The combined bottom flange is unique to box girders. In order to maximize web spacing while minimizing bottom flange width, office practice is to place webs out of plumb on a slope of 1 in 4. Wide plates present two difficulties: excessive material between shop splices and buckling behavior in compression zones (interior piers). To keep weight and plate thickness within reason, it is often necessary to stiffen the bottom flange in compression with longitudinal stiffeners. Office practice is to use tee sections for longitudinal stiffeners and channel bracing at crossframe locations (transverse stiffeners). If possible, bottom flange stiffeners are terminated at field splices. Otherwise, carefully ground weld terminations are needed in tension regions with high stress range. Due to the transverse flexibility of thin wide plates, stiffener plates are welded across the bottom flange at crossframe locations, combined with web vertical stiffeners. For the design of the bottom flange in compression, see LRFD articles 6.11.8.2 and 6.11.11.2.

6.2.4 Special Live Load Criteria for Curved Girders

Refer to AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges 2003, Chapter 3.5. Use HS-25 loading for strength and overload conditions. Use HS-20 loading for deflection checks.

Check live load plus impact deflections on a per bridge basis. Include the overturning effect of centrifugal force. The number of lanes, shall be calculated by rounding down to the whole number of 12 foot lanes that fit on the bridge. Include multiple lane reductions.

Impact is defined in AASHTO Guide Specifications 3.5.6.

Fatigue live load is defined in AASHTO LRFD Guide Specifications 3.5.7 and follows AASHTO LRFD Article 3.6.1.4. The AASHTO LRFD fatigue truck is a fixed axle HS-20 truck, positioned in accordance with 3.5.7.2 of the Guide Specification.

6.3 Design of I-Girders

6.3.1 Limit States for LRFD

Structural components shall be proportioned to satisfy the requirements of strength, extreme event, service, and fatigue limit states as outlined in LRFD articles 1.3.2 and 6.5.

Service limit states are included in Service I and Service II load combinations. Service I load combination is used to check the live load deflection limitations of LRFD article 2.5.2.6. Service II load combination is the LRFD equivalent of the LFD overload provisions. Service II places limits on permanent deflection no yielding, slenderness of the web in compression, and slip of bolted connections.

The fatigue live load specified in LRFD article 3.6.1.4 shall be used for checking girder details per article 6.6. A single fatigue truck, without lane loading or variable axle spacing, is placed for maximum and minimum effect to a detail under investigation. The impact is 15%, regardless of span length. The load factor is 0.75. It is generally possible to meet the constant amplitude fatigue limit (CAFL) requirement for details with good fatigue performance. Limiting twice the calculated fatigue range to the CAFL ensures infinite fatigue life. Webs shall be checked for fatigue loading in accordance with LRFD article 6.10.5.3, using twice the calculated fatigue stress range for flexure or shear. Shear connector spacing shall be according to article 6.10.10. Generally, the fatigue resistance (Z_r) for 7/8" diameter shear connectors can be taken as 2.1 kips for an infinite number of cycles (CAFL = 4.2 kips).

Flanges and webs must meet strength limit state requirements for both construction and final phases. Constructibility requirements for flanges and webs are covered in article 6.10.3. Flexure resistance is specified in articles 6.10.7 and 6.10.8; shear resistance is specified in article 6.10.9

Pier crossframes shall be designed for seismic loading, extreme event load combination. Bolts are treated as bearing type connections with article 6.5.4.2 resistance factors. The resistance factor for all other members is 1.0 at extreme limit state.

6.3.2 Composite Section

Live load plus impact is applied to the transformed composite section using E_s/E_c , commonly denoted n . Long-term loading (dead load of barriers, signs, luminaries, overlays, etc.) is applied to the transformed composite section using $3n$. Positive moments are applied to these composite sections accordingly, both for service and strength limit states. The slab may be considered effective in negative moment regions provided tensile stresses in the deck are below the modulus of rupture. This is generally possible for Service I load combination and fatigue analysis. For strength limit state loadings, the composite section includes longitudinal reinforcing while the deck is ignored.

6.3.3 Flanges

Flange thickness is limited to 4" maximum, but the desirable maximum is 3 1/2". The number of plate thicknesses used for a given project should be kept to a minimum. Generally, the bottom flange should be wider than the top flange. Flange width changes should be made at bolted field splices. Thickness transitions are best done at welded splices. LRFD article 6.13.6.1.5 requires fill plates at bolted splices to be developed, if thicker than 1/4". Since this requires a significant increase in the number of bolts for thick fill plates, keeping the thickness transition 1/4" or less by widening pier segment flanges can be a better solution. Between field splices, flange width should be kept constant.

6.3.4 Webs

Maintain constant web thickness throughout the structure. If different web thickness is needed, the transition should be at a welded splice. Horizontal web splices are not needed unless web height exceeds 14 feet. Vertical web splices for girders should be shown on the plans in an elevation view with additional splices made optional to the fabricator. All welded web splices on exterior faces of exterior girders and in tension zones elsewhere shall be ground smooth. Web splices of interior girders need not be ground in compression zones.

6.3.5 Transverse Stiffeners

These stiffeners shall be used in pairs at crossframe locations on interior girders and on the inside of webs of exterior girders. They shall be welded to the top flange, bottom flange and web at these locations. This detail is considered fatigue category C' for longitudinal flange stress. Stiffeners used between crossframes shall be located on one side of the web, welded to the compression flange, and cut short of the tension flange. Stiffeners located between crossframes in regions of stress reversal shall be welded to one side of the web and cut short of both flanges. Alternatively, they may be welded to both flanges if fatigue Category C' is checked. Transverse stiffeners may be dropped when not needed for strength. If crossframe spacing is less than 3 times the web depth, additional stiffeners may only be necessary near piers.

Stiffened webs require end panels to anchor the first tension field. If jacking stiffeners are placed next to bearing stiffeners, the space created may be used as the anchor panel. Otherwise, the first transverse stiffener is placed no greater than 1.5 times the web depth from the bearing stiffener.

Transverse stiffeners must be designed and detailed to meet LRFD article 6.10.11.1. Where they are used to connect crossframes, they should be a minimum width of 8" to accommodate two bolt rows.

6.3.6 Longitudinal Stiffeners

On long spans where web depths exceed 10 feet, comparative cost evaluations shall be made to determine whether the use of longitudinal stiffeners will be economical. The use of longitudinal stiffeners may be economical on webs 10 feet deep or greater. Weld terminations for longitudinal stiffeners are fatigue prone details. Longitudinal stiffener plates should be continuous, splices being made with full penetration welds before being attached to webs. Transverse stiffeners should be pieced to allow passage of longitudinal stiffeners.

Design of longitudinal stiffeners is covered by LRFD article 6.10.11.3.

6.3.7 Bearing Stiffeners

Stiffeners are required at all bearings to enable the reaction to be transmitted from the web to the bearing. These stiffeners are designated as columns, therefore, must be vertical under total dead load. The connection of the bearing stiffener to the girder bottom flange consists of full penetration groove welds to the bottom flange and fillet welding to the top flange and web. Alternatively, the bottom bearing connection may be made with mill-to-bear and fillet welds or partial penetration welds.

Pier crossframes may transfer large seismic lateral loads through top and bottom connections. Weld size must be designed to ensure adequate load path from deck and crossframes into bearings.

Design of bearing stiffeners is covered by LRFD article 6.10.11.2.

6.3.8 Crossframes

The primary function of intermediate crossframes is to provide stability to individual girders or flanges. Crossframes or diaphragms are required at each support to brace girders; they should be as near to full-depth as practical. Crossframes share live load distribution between girders with the concrete deck. The approximate LRFD live load distribution factors were based on the absence of intermediate crossframes. Where crossframes are present, the exterior girder distribution factors are also determined according to LRFD article 4.6.2.2.2d (conventional approximation for loads on piles). On curved bridges, the crossframes also resist twisting of the superstructure. Pier crossframes are subjected to lateral loads from wind, earthquake, and curvature. These forces are transmitted from the roadway slab to the bearings by way of the pier crossframes. Intermediate crossframes also resist wind load to the lower half of the girders. The primary load path for wind is the concrete deck and pier diaphragms. Wind load on the bottom flange is shed incrementally to the deck through intermediate crossframes. The essential function, however, is to brace the compression flanges for all stages of construction and service life. As such, continuous span girders require bottom flange bracing near interior supports. Office practice requires intermediate crossframes, at spacing consistent with design assumptions. The 25 foot maximum spacing of older specifications is not maintained in the LRFD code. A rectangular grid of girders and crossframes is not significantly stiff laterally before the deck is cured. Both wind and deck placement can cause noticeable deflections. In the case of deck placement, permanent sideway and rotation of the steel framing may result. Some form of temporary or permanent lateral bracing is therefore required.

Crossframes and connections should be detailed for repetitive fabrication, adjustment in the field, and openness for inspection and painting. Avoid back-to-back angles separated by gusset plates. These are difficult to repaint. Crossframes are generally patterned as K-frames or as X-frames. Typically the configuration selected is based on the most efficient geometry. The diagonals should closely approach a slope of 1:1 or 45°. Avoid conflicts with utilities passing between the girders. Detailing of crossframes should follow guidelines of economical steel bridge details promoted by the National Steel Bridge Alliance. Office practice is to bolt rather than weld individual pieces, to provide some field adjustment. Oversize holes are not allowed in crossframe connections if girders are curved.

Intermediate crossframes for straight girders with little or no skew should be designed as secondary members. Choose members that meet minimum slenderness requirements and design connections only for anticipated loads, not for 75 percent strength of member.

In general, crossframes should be installed parallel to piers for skew angles of 0° to 20°. For greater skew angles, other arrangements may be used. Consult with the design unit supervisor or the steel specialist for special requirements.

Intermediate crossframes for curved I-girders require special consideration. Curved girder systems should be designed according to AASHTO "Guide Specifications for Horizontally Curved Highway Bridges." Crossframes for curved girder bridges are main load carrying members and tension components should be so designated in the plans. For highly curved systems, it is more efficient to keep members and connections concentric, as live loads can be significant. Welded connections should be carefully evaluated for fatigue.

Web stiffeners at crossframes shall be welded to top and bottom flanges. This practice minimizes out-of-plane bending of the girder web.

Bridge widening requires special attention to girder stability during slab placement. Lateral movement and rotation has been common with widening projects around the country. Narrow framing, such as a two girder widening, requires bracing to an existing structure. A common method for bracing is to install crossframes (in bay between existing and new girders) with only enough bolts installed to allow for differential deflection but no rotation. Remaining bolts can be installed through field-drilled holes after the slab has cured.

6.3.9 Bottom Laterals

Bottom lateral systems are expensive to install permanently. If possible, they should be avoided in favor of alternative bracing methods. They are seldom required in the completed structure, but may contribute to nuisance or dramatic fatigue cracking in the main girders.

The primary function of a bottom lateral system is to stabilize the girders against lateral loads and translation before the deck hardens. The layout pattern is based on number of girder lines, girder spacing, and crossframe spacing. Cost considerations should include geometry, repetition, number, and size of connections. If used, limit bottom laterals to one or two bays.

For both straight and curved structures, bottom laterals carry dead and live loads, in proportion to distance from the neutral axis. They should be modeled in the structure to determine the actual forces the members experience. Since they carry slab dead load, they should be accounted for when calculating camber.

Where lateral gusset plates are fillet welded to girder webs, the fatigue stress range in the girder is limited to Category E without transition radius, or Category D with carefully made transition radius. The gusset plates should be bolted to the girder web in regions of high tension stress range.

For widening projects, bottom laterals are not needed since new can be braced against existing construction. Framing which is adequately braced should not require bottom laterals.

6.3.10 Bolted Field Splice for Girders

Office practice is to use bolted field splices. Splices are usually located at the dead load inflection point to minimize the design bending moment. See LRFD articles 6.13.2 and 6.13.6.1 for bolted splice design requirements. A method for designing web splices is now outlined in LRFD article 6.13.6.1.4b. Bolted web splices should not involve thin fill material. Thickness transitions for webs, if needed, should be done with welded shop splices.

Flange splice design is outlined in LRFD article 6.13.6.1.4c. For splice plates thicker than 3/4", threads may be excluded from all shear planes for a 26% increase in strength, per article 6.13.2.7. For this to apply, threads must be clear of the nearest shear plane by two thread pitch lengths. Generally, bolts in girder field splices may be designed for double shear.

A new requirement has been added for developing fillers used in bolted splices, article 6.13.6.1.5. When fill plates are greater than 1/4", the splice or filler needs to be extended for additional bolts. As filler thickness increases, the shear resistance of bolts decreases. A way of minimizing filler thickness is to transition flange width for pier segments. Using equal plate thickness by this method has the added benefit of reducing the number of plate sizes in a project.

Splice bolts shall be checked for Strength load combinations and slip at Service II load combination. When faying surfaces are blasted and primed with inorganic zinc paint, a Class B surface condition is assumed.

Fabrication of girder splices is covered by WSDOT Standard Specification 6-03.3(27) and 6-03.3(28). Method of field assembly is covered by section 6-03.3(32) and bolting inspection and installation by section 6-03.3(33). Since bolted joints have some play due to differences in bolt diameter and hole size, field splices are drilled while segments are set in proper alignment in the shop. The joint is pinned (pin diameter equals hole size to prohibit movement) for shop assembly and also during initial field fit-up. Normally, this ensures repeatability of joint alignment from shop to field.

6.3.11 Camber

Camber includes effects of profile grade, superelevation, anticipated dead load deflections, and slab shrinkage (if measurable). Permanent girder deflections shall be shown in the contract plans in the form of camber diagrams and tables. Dead load deflections are due to steel self-weight, slab dead load, and superimposed dead loads such as overlay, sidewalks, and barriers. Since fabricated camber and girder erection have inherent variability, slab form height is adjusted after steel has been set. Although a constant distance from top of web to top of deck is assumed, this will vary along the girders. Deck forms without adjustment for height are not allowed. Girders must be profiled once fully erected, and before deck forms are installed. See *Standard Specification* 6-03.3(39).

Girder camber is established at three stages of construction. First, girder webs are cut from plates so that the completed girder segment will assume the shape of reverse dead load deflections superimposed on profile grade. Only minor heat corrections may be made in the shop to meet the camber tolerance of the Bridge Welding Code AWS D1.5 chapter 3.5. Camber for plate girders is not induced by mechanical force. The fabricated girder segment will incorporate the as-cut web shape and minor amounts of welding distortion. Next, the girder segments are brought together for shop assembly. Field splices are drilled as the segments are placed in position to fit profile grade plus total dead load deflection (no load condition). Finally, the segments are erected, sometimes with supports at field splices. There may be slight angle changes at field splices, resulting in altered girder profiles. Errors at mid-span can be between one to two inches at this stage.

The following is a general outline for calculating camber and is based on girders having shear studs the full length of the bridge.

Two curves will be required, one for total dead load plus slab shrinkage and one for steel framing self-weight. The difference between these curves is used to set deck forms and adjust screed rails.

Girder dead load deflection is determined by using various computer programs. Many steel girder design programs incorporate camber calculation. Girder self-weight is assumed to include the basic section plus stiffeners, crossframes, welds, shear studs, etc. These items may be accounted for by adding an appropriate percentage of basic section weight (15% is a good rule of thumb). Total dead load camber shall consist of deflection due to:

- A. Steel weight, applied to steel section.
- B. Slab weight, applied to steel section. This should be the majority of dead load deflection.
- C. Traffic barriers, sidewalks, and overlays, applied to long-term composite section using 3n.
- D. Slab shrinkage (if $\geq \frac{3}{4}$ ").

Slab dead load deflection will require the designer to exercise some judgment concerning degree of analysis. A two or three span bridge of regular proportions, for example, should not require a rigorous analysis. The slab may be assumed to be placed instantaneously on the steel section only. Generally, due to creep, deflections and stresses slowly assume a state consistent with instantaneous slab placement. For unusual girder arrangements, and especially structures with in-span hinges, an analysis coupled with a slab pour sequence may be justified. This would require an incremental analysis where previous slab pours are treated as composite sections (using a modulus of elasticity for concrete based on age at time of second pour) and successive slab pours are added on noncomposite sections. Each slab pour requires a separate deflection analysis. The total effect of slab construction is the superposition of each slab pour.

Traffic barriers, sidewalks, overlays, and other items constructed after the slab pour should be analyzed as if applied to the long-term composite section full length of the bridge. The modulus of elasticity of the slab concrete should be reduced to one third of its short term value. For example, if $f'_c = 4000$ psi, then use a value of $n = 24$.

Slab shrinkage has a varying degree of effect on superstructure deflections. The designer must use some judgment in evaluating this effect on camber. Slab shrinkage should be the smallest portion of the total camber. It has greater influence on shallower girder sections, say rolled beams. Simple spans will see more effect than continuous spans. For medium to long span continuous girders (spans over 200 feet without any in-span hinges), slab shrinkage deflection can be ignored. For simple span girders between 150 and 250 feet, the deflection should not exceed 1". For calculation, apply a shrinkage strain of about 0.0002 to the long-term composite section using $3n$.

In addition to girder deflections, show girder rotations at bearing stiffeners. This will allow shop plan detailers to compensate for rotations so that bearing stiffeners will be vertical in their final position.

Camber tolerance is governed by the Bridge Welding Code AWS D1.5, chapter 3.5. A note of clarification is added to the plan camber diagram: "For the purpose of measuring camber tolerance during shop assembly, assume top flanges are embedded in concrete without a designed haunch." This allows a high or low deviation from the theoretical curve. In the past, no negative camber tolerance was allowed.

6.3.12 Roadway Slab Placement Sequence

The roadway slab is placed in a prescribed sequence allowing the concrete in each segment to shrink with minor influence on other segments. Negative moment regions (segments over interior piers) must be placed after positive moment regions have had time to cure. This helps minimize shrinkage cracking and provides manageable volumes of concrete for a work shift.

Positive moment regions should be placed first, while negative moment regions are placed last. Successive segments should not be placed until previous segments attain strengths of about 2000 psi or cure 3 to 7 days. This general guideline is sufficient for typical, well balanced span arrangements. For unbalanced span arrangements, the designer should check slab tensile stresses imposed on adjoining span segments. Required concrete strength can be increased, but needless delays waiting for higher strengths should be avoided. Also, the contractor should be given the option of placing positive moment segments with little influence on each other at a convenient rate, regardless of curing time. That is, segments separated by a span could be placed the same or next day without any harm. These can be lumped in the same pour sequence.

6.3.13 Bridge Bearings for Steel Girders

Make bearing selection consistent with required motions and capacities in the following order of preference, high to low:

- No bearings (integral abutments or piers)
- Elastomeric bearings
- Fabric pad bearings
- Steel cylindrical (pin) bearings
- Disk bearings
- Spherical bearings

6.3.14 Surface Roughness and Hardness

The standard measure of surface roughness is the microinch value. Surface roughness shall be shown on the plans for all surfaces for which machining is required unless covered by the Standard Specifications or Special Provisions. Consult *Machinery's Handbook* for common machining practice. Edge finishing for steel girders is covered in *Standard Specification* 6-03.3(14). Surface hardness of thermal cut girder flanges is also controlled.

Following is a brief description of some finishes:

1000	A surface produced by thermal cutting
500	A rough surface finish typical of "as rolled" sections. Suitable for surfaces that do not contact other parts and for bearing plates on grout.
250	A fairly smooth surface. Suitable for connections and surfaces not in moving contact with other surfaces. This finish is typical of ground edges in tension zones of flanges.
125	A fine machine finish resulting from careful machine work using high speeds and taking light cuts. It may be produced by all methods of direct machining under proper conditions. Suitable for steel to steel bearing or rotational surfaces including rockers and pins.
63	A smooth machine finish suitable for high stress steel to steel bearing surfaces including roller bearings on bed plates.
32	An extremely fine machine finish suitable for steel sliding parts. This surface is generally produced by grinding.
16	A very smooth, very fine surface only used on high stress sliding bearings. This surface is generally produced by polishing.

For examples, see Figure 6-4.

For stainless steel sliding surfaces, specify a #8 mirror finish. This is a different method of measurement and reflects industry standards for polishing. No units are implied.

Surface Finish Examples

Figure 6-4

6.3.15 Welding

All structural steel and rebar welding shall be in accordance with the *Standard Specifications*, amendments thereto and the special provisions. The *Standard Specifications* currently calls for welding structural steel according to the AASHTO/AWS D1.5-2002 *Bridge Welding Code* (BWC) and the latest edition of the AWS D1.1 *Structural Weld Code*. The designers should be especially aware of current amendments to the following sections of the *Standard Specifications*, 6-03.3(25) Welding and Repair Welding and 6-03.3(25)A Welding Inspection.

Exceptions to both codes and additional requirements are shown in the *Standard Specifications* and the special provisions.

Standard symbols for welding, brazing, and nondestructive examination can be found in the ANSI/AWS A 2.4 by that name. This publication is a very good reference for definitions of abbreviations and acronyms related to welding.

The designer must consider the limits of allowable fatigue stress, specified for the various welds used to connect the main load carrying members of a steel structure. See LRFD article 6.6. Most plate girder framing can be detailed in a way that provides fatigue category C or better.

The minimum fillet weld size shall be as shown in the following table. Weld size is determined by the thicker of the two parts joined unless a larger size is required by calculated stress. The weld size need not exceed the thickness of the thinner part joined.

Base Metal Thickness of Thicker Part Joined	Minimum Size of Fillet Weld
To $\frac{3}{4}$ " inclusive	$\frac{1}{4}$ "
Over $\frac{3}{4}$ "	$\frac{5}{16}$ "

In general, the maximum size fillet weld which may be made with a single pass is 5/16 inch for submerged arc (SAW), gas metal arc (GMAW), and flux-cored arc welding (FCAW) processes. The maximum size fillet weld made in a single pass is 1/4 inch for the shielded metal arc welding (SMAW) process.

The major difference between AWS D1.1 and D1.5 is the welding process qualification. The only process deemed prequalified in D1.5 is shielded metal arc (SMAW). All others must be qualified by test. Qualification of AASHTO M 270 grade 50W (ASTM A709 grade 50W) in Section 5 of D1.5 qualifies the welding of all AASHTO approved steels with a minimum specified yield of 50 ksi or less. Bridge fabricators generally qualify to M 270 grade 50W (A709 grade 50W).

All bridge welding procedure specifications (WPS) submitted for approval shall be accompanied by a procedure qualification record (PQR), a record of test specimens examination and approval except for SMAW prequalified. Some handy reference aids in checking WPS in addition to PQR are:

Matching filler metal requirements are found in BWC Section 4.

Prequalified joints are found in BWC Section 2.

AWS electrode specifications and classifications are obtained from the structural steel specialist. Many electrode specification sheets may be found online.

Lincoln Electric Arc Welding Handbook.

Many of Lincoln Electric's published materials and literature are available through those designers and supervisors who have attended Lincoln Electric's weld design seminars.

WSDOT *Standard Specifications* for preheat and interpass temperatures.

Notes: Electrogas and electroslag welding processes are not allowed in WSDOT work. Narrow gap improved electroslag welding will be allowed on a case-by-case basis.

Often in the rehabilitation of existing steel structures, it is desirable to weld, in some form, to the in-place structural steel. Often it is not possible to determine from the original contract documents whether or not the existing steel contains high or low carbon content and carbon equivalence. Small coupons from the steel can be taken for a chemical analysis. Labs are available in the Seattle and Portland areas that will do this service quickly. Suitable weld procedures can be prepared once the chemical content is measured.

6.3.16 Shop Assembly

In most cases, a simple progressive longitudinal shop assembly is sufficient to ensure proper fit of subsections, field splices, and crossframe connections, etc., in the field. Due to geometric complexity of some structures, progressive transverse assembly, in combination with progressive longitudinal assembly may be desirable. The designer shall consult with the supervisor and the steel specialist to determine the extent of shop assembly and clarification of *Standard Specifications* section 6-03.3(28)A. If other than line girder progressive assembly is required, the method must be included by special provision. High skews or curved girders should be done with some form of transverse and longitudinal assembly. Complex curved and skewed box girder framing should be done with full transverse progressive assembly. For transverse assembly,

specify crossframe and pier diaphragm connections to be completed while assembled.

During shop assembly, girder segments are blocked or supported in the no-load condition (no gravity effects). Simple line girder assembly is often done in the horizontal position. The primary reason for shop assembly is to ensure correct alignment for girder field splices. For straight bridges, crossframe connections are normally done by numerically controlled (NC) drilling (no trial shop assembly). This is generally of sufficient accuracy to allow crossframe installation in the field without corrective action such as reaming.

For curved I-girders, crossframes are to be fabricated to fit the no-load condition. During field erection, girder segments will need to be adjusted or supported to make fit-up possible. This is not unreasonable since curved girders are not self-supporting before crossframes are in place. However, the method results in out-of-plumb girders. For most cases, making theoretical compensation to arrive at plumb in final condition is not justified.

Highly skewed girders present difficult fit-up conditions. Setting screeds is also complicated because of differential deflections between neighboring girders. Design of crossframes and pier diaphragms must take into account twist and rotations of webs during construction. Often, slotted holes for crossframe connections can be used to allow settlement without undue web distortion. This situation should be carefully studied by grid or finite element analysis to determine amount and type of movement anticipated during construction. Details should be consistent. Unlike curved girders rotating away from plumb at midspan, girder webs for skewed construction should be kept plumb at piers.

6.4 Plan Details

6.4.1 General

Detailing practice should follow industry standards. Designations for structural steel can be found in Table 2-1 of AISC *Detailing for Steel Construction*. Previous plans are a good reference for detailing practices. Detailing should also conform to national unified guidelines published by AASHTO/NSBA Steel Bridge Collaboration.

Details for plate girders are continually being revised or improved to keep up with changing fabrication practice, labor and material costs, and understanding of fatigue behavior. Uses and demands for steel girder bridges are also changing. Cost benefits for individual details vary from shop to shop and even from time to time. For these reasons, previous plan details can be guides but should not be considered standards. Options should be made available to accommodate all prospective fabricators. For example, small shops prefer shorter, lighter girder segments. Some shops are able to purchase and handle plates over 90 feet long. Large shop assembly may be prohibitive for fabricators without adequate space.

In general, office practice is to favor field bolted as opposed to field welded connections. In addition, members of cross frames are shop bolted to give some degree of field adjustment. Welded assemblies have little adjustment during erection.

6.4.2 Structural Steel Notes

Due to their dynamic nature, the structural steel notes are not shown in this manual. They are available as standards in the drafting system. Since each project has unique requirements, these notes should be edited accordingly. Material specifications are constantly changing. Separate sets of notes are available for I and box girders.

6.4.3 Framing Plan

The Framing Plan gives plan locations of girders, crossframes, and attachments. Show ties between the survey line, girder lines, backs of pavement seats, and centerlines of piers. Locate panel points (crossframe locations). Show general arrangement of bottom laterals. Provide geometry, bearing lines, and transverse intermediate stiffener locations. Show field splice locations. Map out different lateral connection details. See Appendix 6-A-1.

6.4.4 Girder Elevation

The Girder Elevation is used to define flanges, webs, and their splice locations. Show shear connector spacing, location, and number across the flange. Locate transverse stiffeners and show where they are cut short of tension flanges. Show the tension regions of the girders for the purpose of ordering plate material, inspection methods (NDE), and *Bridge Welding Code* acceptance criteria. See Charpy V-notch testing requirements of the *Standard Specifications*. Identify tension welded butt splices for which radiographic examination (RT) is required. See *Standard Specifications 6-03.3(25)A*. V and X are also defined in the Structural Steel Notes. Permissible welded web splices should be shown, however, the optional welded web splice shown on the Girder Details sheet permits the fabricator to add splices subject to approval by the engineer. If there are fracture critical components, they must be clearly identified along with CVN call-outs. See Appendix 6-A-2.

6.4.5 Typical Girder Details

One or two plan sheets should be devoted to showing typical details to be used throughout the girders. Such details include the weld details, various stiffener plates and weld connections, locations of optional web splices, and drip plate details. Include field splice details here if only one type of splice will suffice for the plans. An entire sheet may be required for bridges with multiple field splice designs. See Appendix 6-A-3 and A4. Note: Do not distinguish between field bolts and shop bolts. A solid bolt symbol will suffice.

Field splices for flanges should accommodate web location tolerance of $\pm 1/4$ " per BWC 3.5.1.5. Allow a minimum of $1/4$ " for out of position web plus $3/8$ " for fillet weld, or a total of $5/8$ " minimum clear between theoretical face of web and edge of splice plate. The bottom flange splice plate should be split to allow moisture to drain (use 4 equal bottom flange splice plates). The fill plate does not need to be split.

Vertical stiffeners used to connect crossframes are generally 8" wide to accommodate two bolt rows. They must be welded to top and bottom flanges to reduce out-of-plane bending of the web. Stiffeners are coped, clipped or cut short a distance between $4t_w$ and $6t_w$ to provide web flexibility, per LRFD article 6.10.11.1.1.

6.4.6 Crossframe Details

Show member sizes, geometrics (work lines and work points), and connection details. Actual lengths of members and dimensions of connections will be determined by the shop plan detailer. Details should incorporate actual conditions such as skew and neighboring members so that geometric conflicts can be avoided. Tee sections are preferred over double angles for easier painting. If double angles are used, leave a minimum of 1 inch between legs and include fillers as needed for stability. Do not detail crossframes that require piecemeal installation. They must be complete subassemblies for field installation. For highly loaded crossframes, such as at piers or between curved girders, consider symmetric sections with little or no eccentricity in the connections. Where possible, allow for repetitive use of crossframe geometrics, especially hole patterns in stiffener connections, regardless of superelevation transitions. See Appendix 6-A-5.

Internal crossframes and top lateral systems for box girders are shop welded, primarily. All connection types should be closely examined for detail conflict and weld access. Clearance between deck forming and top lateral members must be considered.

6.4.7 Camber Diagram and Bearing Stiffener Rotation

Camber curves should be detailed using conventional practices. Dimensions should be given at tenth points. Dimensions may also be given at crossframe locations, which may be more useful in the field. In order to place bearing stiffeners in the vertical position after slab placement, it is necessary to show expected girder rotations at piers. See Appendix 6-A-6.

Office practice is to show deflection camber only. Geometric camber for profile grade and superelevation will be calculated by the shop detailer from highway alignment shown on the Layout sheets.

6.4.8 Roadway Slab

New bridge decks for steel stringers shall use Deck Protection System 1. Currently, this is a minimum $7 \frac{1}{2}$ " thickness deck, with $2 \frac{1}{2}$ " concrete cover over epoxy coated top mat reinforcing. Allowance for future overlay is not required for this system.

The roadway slab is detailed in section and plan views. For continuous spans, add a section showing negative moment longitudinal reinforcing. If possible, continue the positive moment region reinforcing pattern from end-to-end of the slab with the negative moment region reinforcing superimposed on it. The plan views should detail typical reinforcing and cutoff locations for negative moment steel. Avoid termination of all negative moment steel at one location. See Appendix 6-A-7 and 6-A-8.

The “pad” dimension for steel girders is treated somewhat differently than for prestressed girders. The pad dimension is assumed to be constant throughout the span length. Ideally, the girder is cambered to compensate for dead loads and vertical curves. However, fabrication and erection tolerances result in considerable deviation from theoretical elevations. The pad dimension is therefore considered only a nominal value and is adjusted as needed along the span once the steel has been erected and profiled. The screed for the slab is to be set to produce correct roadway profile. The plans should reference this procedure contained in *Standard Specification* 6-03.3(39). The pad dimension is to be noted as nominal. As a general rule of thumb, use 11” for short span bridges (spans less than 150’), 12” for short to medium span bridges (150’ to 180’), 13” for medium spans (180’ to 220’) and 14” to 15” for long spans (over 220’). These figures are only approximate. Use good engineering judgment when detailing this dimension.

6.4.9 Handrail Details and Inspection Access

If required, include handrails with typical girder details. Locations may be adjusted to avoid conflicts with other details such as large gusset plates. Handrail use shall be coordinated with the Bridge Preservation Office. Often, handrails are not needed if access to all details is possible from under bridge inspection trucks (UBIT’s). Also, easy public access to girder ends and handrails may represent a nuisance. Examine the bridge and site to determine the need for handrails. Fences may be required to deny public access.

Box girders require special consideration for inspection access. Access holes or hatches shall be detailed to exclude birds and the public. They shall be positioned where ladders, as a minimum, are required to gain access. Openings through girder ends are preferred if space behind end walls permits. Bottom flange hatches are difficult to operate. Hatches through webs may reduce shear capacity but are easier to use. Pier diaphragms will require openings for easy passage. To facilitate inspection, interior paint should be light color; white is preferred. Electric lighting and outlets are also required for inspection and maintenance. One-way inspection of all interior spaces should be made possible by round trip in adjoining girders. This requires some form of walkway between boxes and hatch operation from both sides. If locks are needed, they must be keyed to one master. Air vents shall be placed along girder webs to allow fresh air to circulate. Refer to previous projects for details.

6.5 Shop Plan Review

Shop plans shall be checked for agreement with the Contract Plans, *Standard Specifications*, and the Special Provisions. The review procedure is described in Section 1.3.5 of this manual. Material specifications shall be checked along with plate sizes.

Welding procedure specifications (WPS) and procedure qualification records (PQR) shall be submitted with shop plans. If not, they shall be requested and received before shop plans are approved.

Most shop plans may be stamped:

“GEOMETRY NOT REVIEWED
BY THE BRIDGE & STRUCTURES OFFICE”

However, the reviewer should verify that lengths, radii, and sizes shown on shop plans are in general agreement with the contract. The effects of profile grade and camber would make exact verification difficult. Some differences in lengths, between top and bottom flange plates for example, are to be expected.

The procedures to follow in the event changes are required or requested by the fabricator can be found in Section 1.3.6 of this manual. In the past, shop plans with acceptable changes have been so noted and stamped:

“STRUCTURALLY ACCEPTABLE, BUT DOES NOT
CONFORM TO THE CONTRACT REQUIREMENTS”

6.99 References

The following publications can provide general guidance for the design of steel structures. Some of this material may be dated and its application should be used with caution.

1. *Highway Structures Design Handbook, Volumes I and II* by AISC Marketing, Inc., formerly by US Steel

This is a detailed design reference for “I” girders and box girders, both straight and curved, utilizing either service load design or load factor design. This reference has good background on steel bridge details, and how to use them. Although calculations have not been updated for LRFD, the general theory is still valuable. Many shortcuts for design or modeling are presented, such as converting lateral systems into idealized thin plates, and the V load method to approximate curved I girder behavior.
2. *Composite Steel Plate Girder Superstructures*, by US Steel

Example tables and charts for complete plate girders, standardized for 34 and 44 ft. roadways and HS-20 loading. Many span arrangements and lengths are presented.
3. *Four LRFD Design Examples of Steel Highway Bridges, Vol. II, Chapter 1B*
Highway Structures Design Handbook, by AISI and NSBA
4. *Steel Structures, Design and Behavior* by Salmon and Johnson

A textbook for steel design, formatted to AISC LRFD method. This is a good reference for structural behavior of steel members or components, in detail that is not practical for codes or other manuals.
5. *Design of Welded Structures* by Omer H. Blodgett.

This publication is quite helpful in the calculation of section properties and the design of individual members. There are sections on bridge girders and many other welded structures. The basics of torsion analysis are included.

6. *Guide to Stability Design Criteria for Metal Structures*, by Theodore V. Galambos
7. *Curved Girder Workshop* produced by the Federal Highway Administration.

This publication is helpful in the design of curved “I” girders and box girders with explanation of the associated lateral flange bending, torsional, and warping stresses. Approximate analysis techniques are provided.
8. *A Fatigue Primer for Structural Engineers*, by John Fisher, Geoffrey L Kulak, and Ian F. C. Smith
9. *Manual of Steel Construction, Load & Resistance Factor Design*, by AISC

The essential reference for rolled shape properties, design tables, and specifications governing steel design and construction.
10. *Machinery’s Handbook*, by Industrial Press

A reference book for the machine shop practice; handy for thread types, machine tolerances and fits, spring design, etc.
11. *Painting of Steel Bridges and Other Structures*, by Clive H. Hare

This is a good reference for paint systems, surface preparation, and relative costs, for both bare and previously painted steel. Explanations of how each paint system works, and comparisons of each on the basis of performance and cost are provided.
12. NCHRP Report 314, *Guidelines for the Use of Weathering Steel in Bridges*

This reference contains detailing information if weathering steel will be used. Protection of concrete surfaces from staining and techniques for providing uniform appearance is provided.